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ROBINSON'S COLUMN FORMULA.

A. L. BUSH AND W. R. ROBERTS, '88.

[Abstract of a Thesis for the degree of B. S. in the School of Civil Engineering.]

In this work the object has been to make an impartial investigation of Prof. S. W. Robinson's rational formula for finding the strength of full sized columns. Its rationality has been investigated by comparing values of the ultimate strength of columns as given by the formula with the results obtained by experiments, and its merits have been compared with the empirical formulæ in most general use. We have been prompted to make this inquiry because of the difference of opinion in regard to the reliability of empirical as compared with rational formulæ, and also because of the questioned rationality of Robinson's formula. The lack of space has restricted this paper to an abstract from the original article. We have given only the investigations for pin-end, built columns. The comparisons were made for flat-end, built columns, and for solid wrought iron columns, but the results, together with a number of tables and plates, have been omitted here.

The over-practical man holds that the dependence to be placed in results is inversely proportional to the scientific principles involved. He says that a general average obtained from a series of experimental results is more trustworthy than any theoretical formula. On the other hand, he who uses experimental results only to guide him in verifying law, and who also recognizes that reliability and truth are such only because they are in accordance with unvarying principles and laws, would assert that any set of experiments cover but a narrow field and that perhaps incompletely, and, therefore, can not be relied upon in establishing any general formula. Let each reader choose for himself to which of these opinions he will adhere; neither his conclusions, nor ours, will invalidate the facts which are set forth in the following pages.

SOURCE OF DATA.—The data, by which the formulæ have been tested, are the results of experiments made by the Ordnance Department of the U. S. government at the Watertown Arsenal, Mass., during the years 1883 and 1884. The ultimate strength is now considered an unimportant matter; the recognition of the significance of Wohler's law has made the elastic limit the grand desideratum. We recognize these facts, yet have used the ultimate strength, for the reason that experimental data on the elastic limit of full sized

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columns are not to be obtained. Moreover, Prof. Robinson, in testing his formula and in comparing it with others, used the ultimate strength.

FORMULÆ.—*(a) Robinson's formula.**

$$\text{Pin ends:}\quad \frac{T}{A}=\frac{t}{1+\frac{Ad^2}{2I}\left[\left(1+\frac{4l^2}{r^2ed^2}\right)^{\frac{1}{2}}-1\right]} \qquad (1)$$

A=the area, in square inches, of the cross section of the column at the middle; T=the ultimate strength of the column, in pounds; t=the modulus of crushing which for shape iron was taken at 40,000 pounds per square inch; d=the distance, in inches, from the center of gravity of the section to the fiber which first ruptures; I=the moment of inertia for an axis at the center of gravity of the section, and at right angles to the plans of the axial curve of the deflected column; l=the length of the column, in inches; E=the co-efficient of elasticity, taken at 28,000,000, as given by Robinson for shape iron; r=the radius of gyration.

(b) Gordon's formulæ with Burr's constants.†

$$\text{Square columns, pin ends:}\quad \frac{T}{A}=\frac{39{,}000}{1+\frac{l^2}{17{,}000r^2}} \qquad (2)$$

$$\text{Built I columns, pin ends:}\quad \frac{T}{A}=\frac{36{,}000}{1+\frac{l^2}{11{,}500\,r^2}} \qquad (3)$$

(c) Euler's formulæ with T. H. Johnson's constants.‡

$$\text{Pin ends—on tangent:}\quad \frac{T}{A}=42{,}000-203\frac{l}{r} \qquad (4)$$

$$\text{Pin ends—on curve:}\quad \frac{T}{A}=\frac{266{,}490{,}000}{\left(\frac{l}{r}\right)^2} \qquad (5)$$

Formula (4) is applicable when $l \div r$ is less than 138; above this limit use formula (5).

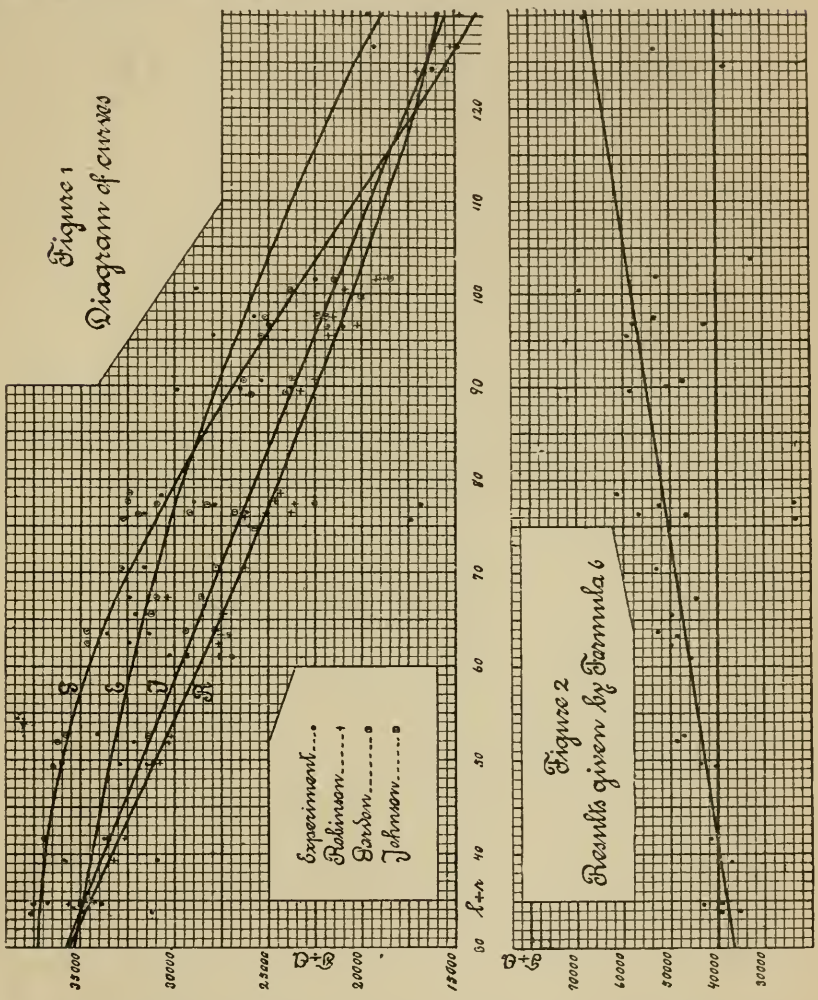
GENERAL DISCUSSION.—Before entering upon the general discussion, attention is called to some irregularities in the experimental results. Thus in Fig. 1, the value, 31,600 pounds, representing the unit strength for $l \div r = 33.8$, was not the ultimate resistance, and for this reason was rejected in platting the experimental curve; similarly for the unit strength, 30,900 of the column $l \div r = 39.1$. The low unit resistance for columns, $l \div r = 76.7$ and $l \div r = 77.2$, was due to the fact that the pins were not in the centers of gravity

* For methods of deduction, etc., see Ohio R. R. Rep. for 1881 and also 1884; also Van Nostrand's Eng. Mag., Vol. 26, p. 490.

† For constants and their application see Burr's Resistance of Materials, p. 441.

‡ For determination and application of constants see Transactions of Am. Soc. of C. Eng., July 1886.

of the cross sections; they also were neglected in platting the experimental curve, marked E. Fig. 1 shows the ultimate strength per square inch as determined by experiment, and also by applying Robinson's, Gordon's, and Johnson's formulæ. The full lines are drawn to show the averages of the several series of valves.



In examining Fig. (1), it is seen that the Robinson curve, marked R, does not conform to that of the experiments, either in direction or position; the curve of the former being concave in the opposite direction to that of the latter. At first sight it would seem that a value of t greater than 40,000 would have given results more

in accordance with the experiments. But reasoning from the effect of such a change with solid wrought iron columns, it is evident that a still greater difference in the *direction* of the two curves would be produced by increasing t. For values of $l \div r$ near 30, the experimental curve begins to assume the direction of the right line, as it should, but Robinson's curve continues concave. The discrepancy mentioned above, due to a change in t, was most marked where both lines should begin to assume the direction of right lines. For values of $l \div r$ between 50 and 120, a change in t would give better results; but this would be introducing empirical values into a rational formula.

It is seen that Gordon's formula gives values considerably too great between 30 and 60, for $l \div r$, and between 100 and 120, they are too small, while the intermediate values lie more closely to the experimental results. However, the general direction of the Gordon curve, marked G, conforms approximately to the experimental curve, marked E. These facts show that an empirical formula is limited in its range of application, and also that the constants used in Gordon's formula are reasonably approximate only for values of $l \div r$ between 60 and 100, and were presumably deduced within these limits.

Johnson's results give a straight line, marked J, for values of $l \div r$ less than 138, as it should. Increasing the constant in his formula from 42,000 to 45,000 would give results nearer the experiments for values of $l \div r$ between 60 and 120, and would not alter the average values for $l \div r$ between 40 and 60. A similar suggestion was made concerning Robinson's formula; by referring to Fig. (1) it will also be seen that the Robinson curve approximates closely to the straight line representing Johnson's formula. It is noticeable that within certain limits of $l \div r$, the empirical formulæ give better results than Robinson's; they are much simpler, and more easily applied. But, for an intelligent application of an empirical formula, a full understanding of the limits of the constants entering into it is necessary; hence, other things being equal, a rational formula is to be preferred before an empirical one.

In Fig. (2), the horizontal line represents the modulus of crushing, t, for built columns, as used in Robinson's formula. The platted results show the value of t, which, if inserted in Robinson's formula, would give the same results as the experiments. These values were determined by the formula below, which was deduced from formula (1) above.

$$t=\frac{T}{A}-\frac{Td^2}{2I}+\frac{T^2l^2d^2}{2r^2eI^2}+\left[\frac{T^2d^4}{4I^2}+\frac{T^3l^2d^2}{r^2eI^2}\left(\frac{T}{A}-\frac{Td^2}{2I}+\frac{T^2l^2d^2}{4r^2eI^2}\right)\right]^{\frac{1}{2}} \quad \ldots\ldots(6)$$

The oblique line in Fig. 2 shows the average value which t should have in Robinson's formula to make it agree with the experiments. Robinson makes t constant, but we see from Fig. (2) that t must vary as $l \div r$, in order that his formula may be made to give an average of the experimental results. According to the theory of the formula,

t should be constant; but the variation mentioned above, is inconsistent with this theory, and must be taken as evidence of a flaw in the analysis by which it was established. Had the value which the horizontal line represents been taken at 50,000, the angle between the two lines would not have been changed, but the average result would have been better; for values of $l \div r$ between 50 and 100 it would have given results very near the experiments.

Although this change would have given a better agreement between the formula and experiments, it would not have brought the two lines parallel, which is but another statement of the variable character of t, and of the irrationality of Robinson's formula. A number of values for the compressive resistance, t, were similarly determined from the data on the solid wrought iron columns and from those on flat-end, built columns. The lines representing these values had the same general direction as the one already discussed, thus further establishing the previous conclusion.

These facts imply the need of some term in the formula varying with $l \div r$. The introduction of some such term might obviate the difficulty of requiring that t should vary.

In this word we had hoped, and preferred, to substantiate a formula based on fundamental principles; but experiment seems to us the court of final appeal, and only when such a formula conforms to experimental evidence can we grant its plea,—rationality.

CABLE STREET RAILWAYS.

WILLIAM BARCLAY, '87, Asst. County Surveyor, Wyandotte, Kansas.

In a busy place like Kansas City, Mo., where everyone is so intent upon business and where time is so valuable, the time consumed in getting to and from different parts of the city makes the question of street railways an important one. Moreover, the streets are, in many places, extremely hilly, making it no small task to go even a few blocks. For these reasons we find the city very generally supplied with means for street transportation, now principally cable lines. It is impracticable to use the horse-cars on account of the extra time consumed and the number of animals required on account of the steep grades.

The first question that presents itself to parties wishing to build a line is that of procuring the necessary franchise. There is more or less difficulty connected with this, which requires considerable time and scheming on the part of the railway projectors. The franchise defines the conditions under which the line is to be operated, and the time within which the work is to be completed. In some cases a sum of money is put up, to be forfeited if the work is not done within the time specified.

The location of the line is determined by the franchise, the established grade of the city being taken, unless the city council may be induced to make a change, which it sometimes does where the the streets have not been improved. The question of grade is not a troublesome one to a cable line, as it can easily overcome any grade that is at all practicable for heavy city teaming.

The first work in the construction of the line is the putting in of a sewer on the center line of the proposed road; this is usually done some time in advance of the other work. Pipe a foot in diameter is commonly used; catch basins for the surface water are built in at intervals as required by the grade. The conduits are drained directly into the sewer. This work has nothing to do with the city system of sewerage. Following the completion of the sewer is the excavation for the cable trench, which is about three feet deep. The depth varies somewhat, as the yokes used are not all of the same size. The yokes, which form the frame work of the road-bed, are placed four feet apart along the line; they are made of heavy cast-iron, and extra excavations must be made to receive them. All material excavated has to be hauled away, since no dirt is used for filling in. The yokes are first placed approximately in position, and the rails and slot-irons bolted to them. This being done, the yokes are brought into their proper positions, and the track is lined up and brought to grade.

The rail in most general use is the "Johnson," which, having a broader base than the ordinary rail, affords a better track for vehicles. Some lines use light railroad rails, weighing about 35 pounds per yard; these are always used where the line crosses a bridge. The slot-irons are bolted to the inner horns of the yokes, so as to leave a three-quarter inch space. On curves this slot must be made about one and a quarter inches wider, to give the grip-shank more room in making the turn. The standard gauge is used, with four feet between tracks, except on curves, where this distance increases as the curve is sharper.

The line is now ready for the concrete, which consists of 5 parts of broken stone, 3 parts of sand, and 1 part of cement. Boards are made to fit between the yokes and conform to the oval opening. The concrete is filled under and around these and thoroughly tamped about the yokes. When the concrete is dry the boards are removed, leaving a smooth, hard surface. Man-holes are left every 35 feet between the slot and outside rail; they are bricked up and covered with wood or iron plates. The pulleys that carry the cables are set in these man-holes. The pavers follow the concrete gang, and pave the space between the tracks. Cedar blocks are generally used. The surface is made slightly concave to carry the surface water to the openings of the sewers. The space between the rails is, in the majority of cases, paved with granite blocks laid in a bed of sand.

This completes the work on the surface; the pulleys along the line, the drums on the curves, and the sheave at the end must next

be put in position. The pulleys, put in at the man-holes before mentioned, are a foot in diameter and about 1½ inches thick. On the curves, drums are used instead of pulleys, which are put in at intervals depending on the degree of curvature. On sharp curves, boards are fitted in between the slot and outside rail so that any part of the cable-way may be exposed. The sheave, which must be built in very solidly, has a diameter equal to the distance between the slots.

A grip-car is now drawn over the line, to see that nothing remains that will interfere with the grip. Then the cable is stretched, the length required having been computed beforehand. The cable is made in one piece, 1¼ inches in diameter. It will sustain a tension of 44 tons. It consists of six strands of nineteen wires each, wound around a hemp center. It is stretched by fastening one end to an extra strong grip, which is drawn over the line by mule power. When in place this cable is spliced by an expert, who unwinds twenty feet from each end and interweaves the strands and makes a splice that can scarcely be detected even by a practiced eye.

The arrangement of the pulleys, so that they will not interfere with the grip, although simple, may not be understood at first by one who has not seen it. On level track, the cable, when in the grip, is carried about fourteen inches below the top of the slot. On summits, the pulleys are placed about eighteen inches below the slot; hence, the grip carries the cable over the pulleys. In depressions, the pulleys are placed about six inches below the slot, and in this case the grip carries the cable below the pulleys. On curves, the drums are set back from the center, and, as the cable is kept to the center by the grip, it is carried away from them. Where two cable lines intersect, the newer one must run its cable under the other, and keep a man at the crossing to hold the upper cable down, so that the grip of the train on the new line may pass over it; the grip on the newer line lets go of its cable while crossing the other.

The cable is wrapped four times around a 12-foot drum at the power end of the line, which allows no slipping; it is kept taut by means of heavy weights, which act on a truck free to move on a track. This truck carries a sheave, around which the cable runs, and which moves with very slight changes in tension.

A man is stationed at a suitable place in the power-house, whose duty it is to watch the cable as it passes before him. Any partial break must be promptly mended. After the grips are taken off for the day, a complete revolution is made, and a closer inspection given the cable. The life of a cable depends largely on the style of the grip, one year being the average length of time that one may be used with safety.

The grip consists of the shank and a pair of jaws. The shank is 16 inches deep, 18 inches wide, and 5-16 inches thick. The jaws consist of two pieces of iron about 15 inches long, which press the cable between them and which may be replaced when worn, and

of horizontal and vertical rollers, which carry the cable when it is not gripped. The cable is never free of the grip except at the end of the line, or when the cable passes under that of another road.

Much inconvenience and many accidents occur on account of buggy wheels dropping through the slot, and engineers are anxious to devise some way for preventing this. One plan suggested is to make the slot at an angle with the vertical, so that a wheel can not go through. The objection to this is that the grip-shank would have to be crooked, which would weaken it, and all the strength that a straight one affords is required. Another plan offered is to use a chain which will fit in grooves left in the slot-irons, and to have a pointed iron run ahead of the grip to raise the chain, carry it up and around the grip-shank, and place it in position again behind.

The brake is a very important part of the cable-car. When the grades are moderate, the ordinary wheel-brakes are used, worked by a lever on the grip car. On very steep grades these are not sufficient, and what is known as the track-brake is used. In this a lever forces two pieces of iron, about 18 inches long and 1½ inches wide, down upon the track between the wheels of the grip-car. In addition to the track-brake, a brakeman rides on the steepest inclines to handle the brakes on the coach. On some lines the gripman tightens his hold on the cable and depends on that to hold the train, but the number of accidents that occur indicates that this is not sufficient. One grade has a rise of 18½ feet in a hundred, and no ordinary brake can hold a car on such a grade.

The usual method of handling cars at the end of the line is to have a switch. Where combination cars, those having the grip and coach combined with a set of trucks under each, are used, a turn-table is necessary. The lines recently built, and also those under construction, have a loop at each end, the tracks branching and running around one or more blocks. In this case a single track is run out one street, around one block, and back.

The cost per mile of constructing and equipping cable railways varies considerably, and the lowest is probably higher than most people would suppose. Figures ranging from $175,000 to $250,000 per mile are given. It costs considerable to maintain the line after it is completed, the pulleys and cable requiring careful attention; and, until the track becomes thoroughly settled, much annoyance is caused by the closing of the slot, which makes it necessary to take up the paving and put the slot-irons in place again. Notwithstanding the cost, one would judge, from the number of new lines that are constantly being put in, that cable lines pay in Kansas City.

CONTOUR MAPS IN LOCATION OF RAILROADS.

E. L. MORSE, '85, Cazenovia, Ill., Resident Engineer, C., B. & Q. R. R.

A wide difference of opinion exists among engineers in regard to the practical use of contour maps in preliminary and location surveys for railroads. While one engineer will consider this part of the topography of prime importance, and will insist on its being taken as accurately, and as widely, as possible; another will do his locating entirely in the field, without the aid of maps, and will make the contour maps only because he is furnished the men for that purpose, and because it is required at headquarters. I quote the following from Prof. Haupt concerning the uses of contour maps: "This method of representing topography is vastly superior to any other, as it exhibits exactly the slope of any portion of the ground; gives the elevation of the base of any object within the tract; enables one to make vertical sections in any direction, with accuracy, from the plot, and to locate roads, paths, or other features upon a given grade, or at any desired elevation; and also furnishes the means of calculating the contents of irregular solids with great precision."

Wellington, in his introduction to The Economic Theory of Railway Location, says: "The instances are almost innumerable, where young men—and old men—of this class have run over, and under, and across a line of the highest operating value, and turned in a costly and miserable line at last. And the contour-map system does not help this evil, even in the hands of a thoroughly capable engineer; for the contour map is simply a device for doing ill in the office, the simplest part of the work, viz: the first approximation to the adjustment of the line in detail; and its most effectual office is to deaden the perceptive faculties of the engineer in charge of the party, and transform him into a mere machine." We find as diverse opinions expressed by locating engineers in the field, as by the authors just quoted.

Improperly used, the contour-map system becomes a "vicious one;" but, if rightly used, it is certainly the most beneficial system. It is of great assistance, especially in a rough country, in enabling the locating engineer to find the best line. In a prairie country, where the ground is comparatively level, where long tangents can be run, and where the line may be shifted a considerable distance either way without making any great change in the earthwork, the contour map is of no great assistance; but where the line is a supported one, confined within narrow limits in location, and requiring the nicest engineering skill to keep it to maximum grade and curvature, and

"get out" at the top, or "down" to the bottom, the contour map can be used to good advantage, and the necessity of backing up to re-run the line can thus be avoided.

It is essential that the man who personally superintends the location shall be the one who makes it on the contour map; for, by means of the former, he becomes familiar with the features of the ground over which the line is run. The contour map is, to him, an "intelligible image of the ground," showing the relation of distant parts to each other. In the field his view is limited to a small part of the line, and he is an exceptional engineer who is able, even after long experience, to judge accurately of the relative elevations of points some distance apart. Nor is he likely to carry in his mind the details of the ground in a day's run. Since a change in one part of the line will always effect some other part, it is necessary to know what the effects of such a change will be. If this is determined in the field, it will be a "cut and try" process, with oftentimes no better results than before; for what is gained in bettering one part of the line is lost by increasing the work in another part, thus making a third and sometimes a fourth location of this part of the line necessary. This involves delay and expense which may be avoided, to a great extent, by an intelligent use of the contour map. With the map before him, the engineer has a comprehensive view of all the details of the part to be changed, and its connection with other parts effected by such a change; being familiar with the main features of the ground, he will understand, at once, what is needed. He will not only be able to tell the effects of a change in the line and where a change can be made to produce good results, but will also be sure that the expected results will be obtained when the line is run in the field. The map will also give him notes of the irregular cross sections from which the earthwork may be computed, and a close estimate of cost can be obtained. All this may be done after supper, in camp, or while the party is in the field at work, without involving loss of time; and may avoid the trouble and expense of re-running the line. The curves may also be fitted in on the map before being staked out in the field, thus determining what degree of curve will best fit the ground, and showing whether the line would be benefited by a change at this point.

I have presupposed the contour map to be a correct one, made on the preliminary survey. Usually the located line will not vary any considerable distance from the preliminary line, so that the contour map made from the latter may be used in the location. Especially is this true of supported and side-hill location, in which cases the contour map is of the greatest value. On one survey, from the Mississippi river out, I remember the locating engineer would make a "proposed location," in pencil, on the preliminary contour maps before sending them in. These maps were blue-printed and joined together, thus giving the locating engineer a continuous contour map of the entire line.

The method used in taking contours in the field is quite simple. Usually the topographer, with one assistant, can do the work and keep up with the party, unless the country is very rough. A hand level, level rod, and tape line are needed; also a book ruled in squares representing fifty feet each way, with a place on the margin for notes of elevation and distance out of contours. The station of the center line, the elevation of which is obtained from the level book, is always to be taken as the starting point. Contours showing a difference of five feet in elevation are sufficient for practical purposes, and often where the line is on a steep side hill, ten feet contours will be all that can be platted on a map of the usual scale. Distances may be obtained by pacing where the slope is gentle; but if the ground is rough, or steep, the tape or rod should be used to measure the distance. The contours should be sketched in by the topographer in the field, especially where the ground is broken and where there are points and hollows to be shown. The sketch can be quickly and accurately made. By means of it the irregularities are shown, and a draughtsman, who has not seen the ground, is better able to understand and correctly plat the notes

The topographer must exercise good judgment in determining where the contours should be taken at a considerable distance from the line, and where the ground should be fully and accurately shown. In fact, I think his position in the party, in point of knowledge and judgment and in importance of work, is next to that of the locating engineer; especially is this true where the contour map is used. He should be able to judge distances and elevations pretty accurately, and to show by sketch the main feature of the country, outside of the limits of the contour lines; and, while this work need not be artistic, it should give a good idea of the lay of the country. Very often, where it is necessary to swing the line from its course, questions and explanations may be avoided by showing on the map the reasons for so doing.

THE DETERMINATION OF WATER-WAY FOR BRIDGES AND CULVERTS.

By A. N. TALBOT, Ass't Professor of Engineering and Mathematics.

[Read May 28, 1887.]

In general, little consideration has been given to the proportioning of the span of culverts and bridges. Boxes, pipes, and small bridges are guessed off by the subordinate engineer. Too often a mistaken size causes a washout of great magnitude, entailing cost of repairs and loss of traffic. Nearly as frequently extravagant ideas of requirements cause greatly increased cost of construction. Indeterminate though the problem be, it demands an intelligent treatment. Any one can make a culvert or bridge large enough; it is the province of the engineer to design one of sufficient but not excessive size. Perhaps in the majority of bridge locations, bluffs or a well-marked channel determines the width of the bridge opening, so that the amount of water is a subordinate feature, or the size of the opening can be decided upon by inspection of the profile. Very often, however, there is no such guide and no good evidence of the height of the stream at the opening. This paper will consider such cases. What I have to offer is not universally applicable—and no rule of thumb will fit the case.

Here let me warn you that the question is not one which can be determined with mathematical exactness. The problem is more or less indeterminate—decimal places do not give accurate results with uncertain data. Do not then be troubled because similar conditions give varying results.

In the construction of a new railroad, considerations of first cost, time, and a lack of knowledge of the amount of future traffic, as well as ignorance of the physical features of the country, usually require that temporary structures be first put in, to be replaced by masonry and iron later. An excessive size for these temporary structures may not be very objectionable—often a trestle will for the time be cheaper than an embankment. With increased resources, permanent improvements will be made. In the meantime an incidental but very important duty of the engineer is to make a careful study of the requirements of the road. Upon the judgment and ability displayed in this depends most of the economical value of the improvements, as the road will have fixed or standard plans for culverts, abutments and piers, and the supervision of the construction is not difficult. High-water mark of streams and the effect of floods even in water-courses ordinarily dry should be recorded and

notes of required water-way taken. With these data the proper proportioning of the permanent structures becomes an easy task.

CULVERTS.

The difficulty of deducing a rational formula to determine the necessary span or water-way of a culvert draining a given area will be appreciated on examining the following conditions which enter into the problem:

(1). The variation of the rate of rainfall in different localities.

(2). Paucity of data, since records are generally given as so much per day and rarely per hour, while the duration of the severe storm is not recorded.

(3). The melting of snow with a heavy rain.

(4). The permeability of the surface of the ground, depending upon the kind of soil, condition of vegetation and cultivation, etc.

(5). The degree of saturation of the ground and the amount of evaporation.

(6). The character and inclination of the surface to the point where the water accumulates in the water-course proper.

(7). The inclination or slope of the water-course to the point considered.

(8). The shape of the area drained and the position of the feeders.

The importance of the last item will be seen in comparing a spoon-shaped area, shown at (*b*) in the accompanying cut, where the main water-course is fed by branches from both sides so arranged that water from the whole area reaches the culvert at the same time, with a long, narrow basin, shown at (*a*), in which before the water from the upper part reaches the opening the rainfall from the lower portion has been carried away and the severe part of the storm is past. In large areas this is an important consideration.

In constructing such a formula, then, we should have to decide between 1 and 3 inches of rainfall per hour; upon the proportion of the rainfall reaching the stream, which is given in reliable data as varying from 15 to 50 per cent., and in case of snow to a quantity even greater than the rainfall; to estimate the time required to find its way through grass and brush, over flat and steep, until the water-course is reached, and then with the imperfect formula for flow in channels to correct for retardation by bends and brush and grass; and finally to decide upon what proportion of the water carried down the stream would reach the opening at the same time. As any of these estimates might be in error 200 or 300 per cent., the final result would not be reliable.

Before deciding upon a method, I desire to emphasize the fact that any formula will be approximate, that the estimation of the values of the different conditions entering into the subject will be almost wholly a matter of judgment, so that the formula must be considered more as a guide to the judgment than as a working rule. Mathematical exactness is neither warranted by the data nor required

by the problem. The question is not one of 10 or 20 per cent. of increase. If a 2-foot pipe is insufficient, a 3-foot pipe will probably be the next size, an increase of 225 per cent. If a 6-foot arch culvert proves too small, an 8-foot will be used. The real question is whether we want an 8-foot culvert or a 2-foot pipe. Approximation is all we can hope for, and fortunately is all we require.

At first thought it would appear that the amount of water to pass through an opening, other things being equal, will be directly proportional to the area drained. For small areas this is nearly true, but as we have seen in the case of the long, narrow drainage area there will be a variation from the direct proportion as the size of the basin increases.

The slope of the culvert is not considered nor is the increased discharge of large openings per square foot of area, as nearly the same proportion will be found as in the channel above.

Maj. E. D. T. Myers' formula is the only one in general use. It is

$$\textit{Area of water-way in square ft.} = C\ \sqrt{\textit{Drainage Area in Acres}},$$

where C is a variable coefficient for which 1 is used in ordinary, slightly rolling prairie or agricultural land, 1.5 in hilly ground, the value rising to 4 in mountainous and rocky ground. This gives values which are ample for small drainage areas, in fact too large. For large areas the results are too small. The defect for small areas is apparent from the fact that the discharge will be then nearly proportional to the acreage, while the formula makes 40 acres require only twice as much as 10 acres. With judicious use, this formula has done good service, but it has many defects.

It has been suggested that observations be made at the culvert to find the depth of water there for a known rainfall. A maximum rate of rainfall could then be assumed, and the maximum water-way calculated by simple proportion. The objection to this method is the fact that a double amount of rainfall will give much more than double the discharge, since the ground has become saturated and since obstructions will have less effect on the flow.

The only formula for finding the discharge for a given rainfall over a given area approaching to reliability is that of the Swiss hydraulician, Mr. Burkli-Ziegler, which was derived from observations on heavy storms in Zurich, together with information from other European localities. For feet measure it is

$$\textit{Quantity of discharge per acre in cubic feet per second} = c\ a \sqrt[4]{\frac{s}{A}}.$$

c is a coefficient depending upon the character of the ground and having 0.31 for an average value; a the rainfall in cubic feet per second per acre, which roughly may be used as 1 cubic foot per second per acre for each 1 inch of rainfall; s the fall per 1,000, and A the area in acres. Proper substitutions in this would give the volume of

flow, from which the necessary water-way could be calculated, but it will be better to use only deductions from this.

Since by this formula the quantity discharged *per acre* varies inversely as the fourth power of the area drained, the volume of discharge *from the whole area* will vary as $\frac{A}{\sqrt[4]{A}}$, or $A^{3/4}$; and, assuming the same velocity through the culvert as in the stream above, the opening will vary likewise. This assumption will be true when the grade of the culvert is the same as that of the stream above and when the smaller coefficient of friction in the culvert over that of the channel itself is counteracted by the resistance to entering the culvert. We may then write

$$a = C \sqrt[4]{A^3}, \text{ or}$$

$$\textit{Area of water-way in sq. ft.} = C \sqrt[4]{\left(\frac{\textit{Drainage area}}{\textit{in acres.}}\right)^3},$$

for which the coefficient C must be determined.

By comparison with the formula of Burkli-Ziegler and with the flood flow of streams up to several of 77 square miles area, I conclude that for rolling agricultural country subject to floods at time of melting of snow, and with the length of valley three or four times the width, $\frac{1}{3}$ is the proper value of C. If the stream is longer in proportion to the area, decrease C. In districts not affected by accumulated snow, and where the length of the valley is several times the width, $\frac{1}{5}$ or $\frac{1}{6}$ or even less may be used. C should be increased for steep side slopes, especially if the upper part of the valley has a much greater fall than the channel at the culvert.

In any case, the judgment must be the main dependence, the formula being a guide to it. On a road already constructed the C may be determined for the character of surface along that line by comparing the formula with high-water mark of a known drainage area. Experience and observation on similar water-courses is the most valuable guide. A knowledge of the action of streams of similar situations in floods and of the effects of peculiar formations and slopes is of far more value than any extended formula.

The drainage area may be obtained with sufficient accuracy from a county map, or by walking up and across the basin.

The following table shows the values as given by the Myers' formula $a = 1\sqrt{A}$ and by $a = \frac{1}{3} A^{3/4}$. a is in square feet in both cases:

ACREAGE.	MYERS.	TALBOT.	ACREAGE.	MYERS.	TALBOT.
10	3.2	1.9	200	14.1	17.7
20	4.5	3.2	400	20 0	29.8
40	6.3	5.3	640	25.3	42.4
80	8.9	8.4	1000	31.6	59.3
100	10	10.5	2000	44.7	99.7
160	12.6	15.0	6400	80.0	23,85

According to the Myers formula a 12-inch pipe will not drain an acre, which is absurd on its face.

Incidentally, I may say that I have found that for average slope in small rivers the flood discharge per square mile is about

$$\frac{500 \text{ cu. ft. per second.}}{\sqrt[4]{\text{Area in sq. miles.}}}$$

BRIDGES.

Ordinarily the distance between banks and the prevailing grade line on either side determines the length of span and the amount of water-way. Large streams having well-established channels will not present any difficulties—an inspection of the profile is the only thing needed. With small rivers and creeks, especially in regions subject to floods and having no well-defined banks and high-water channel, a more extended investigation is necessary. The requisite water-way should be determined before the permanent iron bridges are constructed.

In brief, the process is to find the flood discharge of the stream and the average flood velocity at the bridge opening. Their quotient will give the necessary area of the opening, from which the width may be determined.

The flood discharge can not be taken by the more accurate method of finding the average velocity of the section by the current meter, for even if the engineer were present at the time of the highest water known, time and other circumstances would not permit the gauging.

The best method is to find the sectional area of the high-water stage, and in connection with the slope calculate the flood discharge.

To this end select several sections above or below the bridge at points where the high-water mark may be certainly and accurately determined and at distances apart depending upon the size of the stream. If a tributary enters between the section and the bridge, its discharge must be determined similarly and applied. Select as straight and uniform a reach as possible, for bends, crooks, obstructions and other irregularities will increase the error of the result. The stream itself may be sounded in a boat rowed in range or secured to a rope stretched across the stream. The remainder of the cross-section may be taken in the usual way of leveling to secure a profile. Care must be taken that the section is at right angles to the current. The height of high water is the most difficult and unsatisfactory detail to be found. The position of drift wood and the evidence of residents will be the principal means. Divergence of opinion and irreconcilable evidence must be sorted out by an exercise of common sense. In any case high water on one bank or in the middle of the stream may vary considerably from that on the other side. From the accepted notes the area of the flood discharge is calculated.

The slope is best determined by taking the elevation of two well-

defined high-water marks, one above and one below the section and at least 1,000 feet apart, provided the reach is straight and of nearly uniform section for that distance; or it may be found that the slope of the stream at its usual stage will give fair results. In case of low water or none, the fall of the bottom of the channel will give the slope approximately. If the cross-section of the channel varies much, neither the slope of the bottom nor of high-water will give the proper inclination.

Various formulas have been proposed for estimating the discharge of a stream or open channel. Most of them are of the form $V = c\sqrt{RS}$, known as the Chezy formula. In this V = velocity in feet per second. S = rate of fall or sine of slope. R = hydraulic mean radius = $\frac{area}{wetted\ perimeter}$. c = a coefficient, formerly thought to be constant, but now known to be variable and to depend upon roughness of channel and change of section of the stream, upon depth, slope and even upon the velocity. In fact the velocity does not vary as the square root of R and S. The formula is based upon the assumption that the resistance to flow varies as the square of the velocity and as the proportion of the water that is in contact with the wetted surface, neither of which is exactly true. Fteley's formula for conduits, $V = c\ R^{.62}\ S^{.5}$ gives close results for the kind of channels it was derived from.

However, the form of the Chezy formula is now generally accepted, and a modification is made in c according to the other conditions. Kutter's formula for this coefficient is

$$C = \frac{41.6 + \frac{1.811}{n} + \frac{.00281}{S}}{1 + \left(41.6 + \frac{.00281}{S}\right)\frac{n}{\sqrt{R}}}$$

where n is a coefficient of resistance of the channel with values varying from .030 to .035 for rivers. Hamilton Smith, jr., a late authority in hydraulics, rejects Kutter's formula as untrustworthy. He further says: "For conduits or canals with rough channel, we regard it as almost useless to propose any other formula than $V = c\ \sqrt{R\ S}$; it is impossible to assign with any reasonable degree of accuracy any numerical value to the resistance of the channel for streams, and as it is such a controlling quantity in any equation of which it forms a part, such uncertainty must always make the equation simply approximate."

For our purpose then it will be best to use the Chezy formula, using the judgment and comparing with known values of c for streams that have been gauged.

The following values have been found by experiment:

Irrigating ditches with rough sides, sharp curves, fall of 15 to 20 feet to the mile and depth of 2 to 4 feet, c = 30 to 50.

Creeks and small streams, with uneven channels and many obstructions, c = 50 to 70.

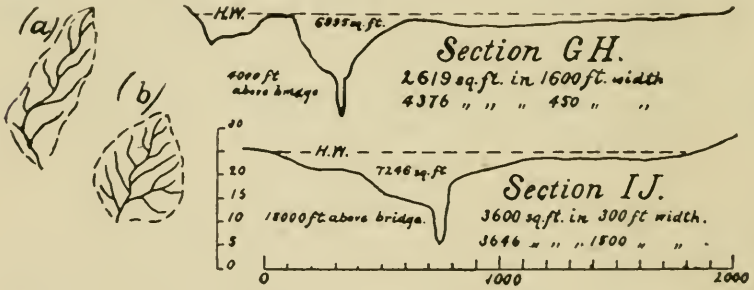
Small rivers with depth of 10 feet or more and width of 100 to 400 feet, $c = 60$ to 90.

Rivers with smooth banks and channel and straight course. $c = 100$. For large rivers this must be increased.

Fortunately, for our purpose the absolute volume of discharge is not necessary, since we shall finally divide the discharge by the velocity at the bridge to get the required area of water-way, and a difference in coefficients will not show a great discrepancy.

Having calculated the velocity, multiply it by the area of cross-section to determine the volume of discharge; average the quantities found from the different sections and get the average velocity. If the situation admits it, one of the sections should be at the bridge opening. By dividing the total discharge (using the average of the different calculations) by the calculated or an assumed velocity at the bridge, the necessary area of water-way is obtained; then, from the depth and cross-section of the channel, the necessary length and height of the opening is found. Large discrepancies may be expected in the results for the different sections.

As an illustration of what I mean, I give the record of surveys of Apishapa river in Colorado. Two cross-sections of the stream,



one 4,000 feet and the other 18,000 feet above the bridge, are given. The banks were overflowed, and the shallowness of the outer section required a smaller c, so in the calculations the cross-sections are divided into two compartments, a shallow and a deep one, the latter including the portion of the section within the principal banks of the stream. Then separate computations are made for each compartment. The hydraulic mean depth, R, is found by dividing the area of either portion of the section by its width. The coefficient c is assumed to be 90 in the deep portion and 70 in the shallow. The results are shown as follows:

SECTION GH.

	c	*Area in sq. ft.*	*R*	*S*	*V*	*Volume.*
Deep Comp.....	90	4,376	9.72	.0018	11.88	51.986
Shallow "	70	2,619	1.64	.0018	3.78	9.900
Total.....	.....	6,995	.....	.....	.....	61,886

Average velocity, 8.85 feet per second.

SECTION IJ.

Deep Comp.....	90	3.600	12.00	.0018	13.23	47,628
Shallow "	70	3,646	2.43	.0018	4.62	16,844
Total.....	.....	7,246	.....	.....	.....	64,472

Average velocity, 8.90 feet per second.
Average volume of discharge, 63,179.

Using the same slope for the section at the bridge opening, the velocity there would be nearly 16. Since at this point there certainly would be back-water from the river, this value was thrown out, and the average of the velocities in the deep compartment of the other sections, 12.5 feet per second, was taken as the average velocity at the bridge opening. Dividing 63,179, the average volume of discharge, by this velocity, gives a necessary water-way of 5055 square feet.

An examination of the profile at the bridge will determine the length of span necessary to secure that water-way. However, in the case referred to, it was decided to excavate the channel from abutment to abutment to the depth of the bottom of the stream. The resulting uniform depth was 25 feet. The area of water-way, 5055, divided by this depth, gives 202 as the necessary width. The span must be increased to allow for the amount taken up by the piers, etc.

Not enough importance is attached to the shape of the channel at and below the bridge opening. Even when the channel is excavated to the full width of the opening under the bridge, the full capacity of the bridge opening will not be utilized unless the enlarged channel is extended down stream until the flood water has a chance to escape over the flat below without a reduction in velocity and a consequent obstruction to the water that is at that time coming through the opening. In fact, the shape of the channel just below the bridge has as much to do with the capacity as the opening itself. Above the bridge, whatever may be the shape of the opening, the pressure of the water on all sides and the tendency to dam up tends to force the water through the opening with increased velocity. Below it, any decreased section or turn in the course obstructs the current and greatly impedes the velocity of water at that time flowing through the opening. The principle is similar to the ajutage with curved form used in the experiments in Physics. Many bridges have come under my observation whose efficiency could be doubled with little expense by modifying the channel in this way.

It was my intention to treat the amount of back-water caused by the contraction of the channel at the bridge and by the obstruction of the piers, but the length of this article requires that that subject be given in a separate paper.

STRENGTHENING IRON RAILROAD BRIDGES.

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As railroad lines increase, competition sharpens, and it becomes necessary to lessen the cost of transporting freight and passengers. Ten to fifteen years ago the weight of a freight car and the weight of its load were about equal. But while the weight of the car has remained nearly the same, the weight of the load has been steadily increased until it has not only been doubled but even trebled; and cars are now carrying thirty tons of freight, which with the weight of the car makes a very heavy load. But of course the weight per lineal foot of the engines hauling such cars is more than the weight per foot of the cars, and one would hardly expect to find the cars producing greater strains in a bridge than the engines. But if we take an eighty-five ton mogul engine, and two thirty-ton capacity freight cars, and compare the strains produced by each in the floor system of a bridge having sixteen-foot panels, we find greater strains produced by the cars than by the engine.† With the increased capacity of cars comes the increased weight of locomotives. With the increased weight of engines and the increased carrying capacity of cars must come the increased strength of bridges. The Pennsylvania Railroad Company requires its bridges to be built to sustain a load consisting of two engines weighing 120 tons coupled together, and followed by a load of two tons to the lineal foot. Their load for the floor system and short span girder is twenty-five per cent. greater than this. Bridges built today are, as a rule, from fifty to seventy-five per cent. stronger than were customary ten or twelve years ago. Hence bridges built then, to be in use today, must have had a great excess of material in them when built, or else be unsafe for the present loads. All bridges have some excess, but it would be unreasonable to suppose that they were made seventy-five per cent. stronger than was considered necessary for the loads they were intended to carry. By comparing the strains as given when the bridge

† The author submitted diagrams comparing the concentrated loads of 30-ton freight cars with those of a 85-ton mogul engine: the trucks of two cars concentrate 90,700 pounds on $15\frac{1}{2}$ feet, and the drive-wheels concentrate 90,000 pounds 14 feet.

was built with the strains due to the present loads, one can easily satisfy himself that there are many bridges that need strengthening or replacing with new ones. To rebuild the bridges on our railroads would necessitate the expenditure of a large sum of money, and for the most part part is needless. For if the bridge can be strengthened even by adding another bridge of the same strength to the one already built, it will be economical to do so. But if it is unnecessary to rebuild the weak bridges, they must be strengthened, perhaps not in all parts, for many of the common iron bridges built ten to fifteen years ago are found upon examination to be strong in some parts and weak in others. Generally the floor system will be found to be the weakest part, owing to the extra strains caused by the concentrated weights on the short wheel base of our present heavy loads.

Formerly engine excess was not considered of sufficient importance to be taken into account in figuring strains in trusses, and even to-day some bridge builders look at it in the same light.

The floor may be strengthened in different ways, depending on the amount of strengthening needed. It may be replaced entirely by a new one, and the old stringers and floor beams used for stringers on small open culverts where wooden ones are now in use. Or, perhaps the floor beams and stringers are I-beams; then by adding other I-beams side by side, we may get any desired strength. Or, if the floor is made up of built beams, a plate can be added to the top and bottom flanges, raising the strength to the shearing strength the rivets and web. The web can be made stronger by putting in stiffners. Or, if adding plates does not furnish sufficient strength, other beams may be placed beside the old ones. These changes may be made without taking the stringers and floor beams from the bridge. Plate girder bridges may be strengthened by the same method as the built beams. Or, if strength enough can not be gained that way, they can be used over shorter openings.

Truss bridges may be strengthened in different ways. What would suit one might not be applicable to another, and so each bridge must be strengthened as will best suit the circumstances. Suppose we have a bridge strong enough in some parts and weak in others. Formerly the machinery for constructing bridges was not as complete as it is at present, and consequently, the parts requiring considerable machine work, as eye-bars, cost more by the pound than did the members made up of plates, channels, angles, etc., which had only to be cut, punched, and riveted. The bridges were built and paid for by the pound; and when the opportunity offered, some bridge companies were not slow about putting in heavier plates and angles than were required, thereby making the compression members proportionally much stronger than the tension members. In a certain truss bridge of one hundred and twenty-four foot span, built in 1878 by a bridge company not now in existence, the compression members would sustain a load fifty-five per cent. heavier than the tension members and pins. After examining the connections and finding

what strains could be taken care of there, it became an easy matter to strengthen the pins by replacing them with steel ones, and strengthening floor beams and stringers by adding plates to the top and bottom flanges. The pins throughout the bridge being of the same diameter, the chord bars and inclines were shifted to different places, and by adding a few extra bars to supply strength where needed, they became as strong as the compression members, and we then have a bridge over fifty per cent. stronger than as originally built, and at a small cost compared with building a new one and the old one into the scrap heap.

Or, perhaps we have at one point on the road a fifty-foot Howe truss that needs rebuilding on account of age, and at some other point a light iron truss of, say, ninety-foot span with panels of about fifteen feet; then by taking out the two panels nearest the center, we have shortened the bridge to a sixty-foot span, and have a bridge that is fifty per cent. stronger than it was before, in every thing except the floor system, and that may be strengthened by one of the methods given above. This bridge can be moved, and replace the Howe truss.

Or, supposing there are two or any other even number of spans of the same length and make of bridge, all of which are light and need strengthening. These spans need not be at the same point but may be miles apart. They can be used as were those of a bridge over the Illinois river which had six spans of one hundred and twenty feet each, one span of one hundred and seventeen feet, and one draw span of three hundred feet, all of which were lighter than now required. As all the members throughout the bridge were light, the first method given above would not suit this case. Having no place to use the draw span, nor the one hundred and seventeen foot span, they were thrown aside. Then three of the one hundred and twenty foot spans were taken down and the trusses placed on the piers with the other three spans of the same length, so that they formed double trusses on each side of the track. The floor beams and stringers also being light were thrown aside and new ones put in, using a built beam for the floor beams and two fifteen-inch I-beams under each rail for stringers. The floor beams were suspended from the lower chord pins, using four suspension hangers at each end of each floor beam, and arranged on a steel equalizer so as to have each truss equally strained. The trusses were two feet between centers. They could not be placed nearer together on account of the size of the pedestals at the end posts. The four trusses are connected at the top by a six-inch I-beam strut and by lateral rods. The spans taken down were replaced with new ones. Very little delay was occasioned to trains, one train only being delayed less than two hours while putting in the center posts and swinging the draw.

THE COEFFICIENT IN THE FORMULA FOR BAROMETRIC LEVELING.

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[Abstract of a Thesis submitted for the Degree of B. S. in the School of Civil Engineering.]

The increasing number of topographical and geological surveys in progress under the direction of the general government, and also of the states, renders some simple and expeditious method of determining elevations absolutely necessary. The barometer affords one of the most ready means of making the observations necessary for this purpose; but, unfortunately, it is not an instrument of great precision, and in surveys where great accuracy is required it is useless. A few observations may determine the elevation approximately, but if a reasonably exact determination is required readings must be taken very often and over a great length of time, thus requiring more time than can be afforded for this part of the survey. Nevertheless barometric leveling is the best means of determining a number of elevations with sufficient accuracy for map-making purposes.

The formulæ employed in this system of leveling have all, with a single exception, been deduced entirely on theoretical grounds. The most important part, by far, of any barometric formula is the coefficient. Heretofore it has been assumed that this constant depends only upon the ratio of the weight of a cubic unit of air to that of mercury. It is the object of this paper to point out some of the errors not provided for in the ordinary method of deducing the formulæ for barometric leveling and also to indicate a method of determining the barometric constant in such a way as to represent the average of the actual conditions.

The ordinary barometric formulæ have corrections for the temperature of the instrument, for the temperature of the column of air, and for difference of latitude; but these corrections cannot wholly eliminate the error caused by these sources. The temperature of the column of air is considered as varying uniformly; but this cannot be, owing to the great local heating that takes place near the surface of the earth. The error caused by the temperature of the instrument, and the instrumental errors are small in a well made barometer carefully handled. The common formulæ have no term for humidity. The errors due to humidity are caused by the lower stratum of air becoming more or less saturated with watery vapor and thereby making it much lighter than dry air.

The greatest source of error is that due to gradient. This is not so connected with the fluctuations of the barometer as to admit of any term in the formulæ for its correction. The gradients are diurnal, annual, and non-periodic.

The diurnal gradient is caused by the difference in the heating effect of the sun between day and night. This gradient reaches its maximum at 9 or 10 a. m.; in the afternoon there is a minimum about 3 or 4 p. m. From the afternoon minimum it rises until about 11 p. m., then falls until 4 a m., when it again rises until 9 or 10 a. m. This is plainly shown in the reports of the U. S. Chief Signal Officer, except for 4 a. m., for which hour no observations are given. For example, for Portland, Me., in the report for 1885, the mean of the twelve months for 7 a. m., 3 p. m. and 11 p. m. are 29.960, 29.916 and 29.936 respectively; for Atlanta, Ga., they are 28.934, 28.892, and 28.921; for St. Louis, 29.455, 29.424, and 29.441. Similar results are shown in 95 per cent. of all the stations examined.

The annual gradient is caused by the movement of the sun from one hemisphere to the other. This is no less clearly shown, in the report mentioned above, than that for the diurnal gradient. In glancing down the column of barometric readings, commencing with January, it will be noticed that in nearly all cases there is a gradual decrease in the readings until the middle months, June and July; then there is a steady rise until December or January. This change is so slow and uniform that it can not very much modify readings taken over a short space of time.

The non-periodic gradient is caused by the variation in the heat of the sun and also by local conditions. The non-periodic gradient may be the source of a very great error, as it is very difficult to know when it exists. In the report referred to above, we find at Mt. Washington on January 27th, a range in the readings of 1.282 inches; for February 16th, 1.527 inches; for March 18th, 1.069; for April 27th, 1.111; for May 22nd, 0.980; for June 15th, 0.621, and similarily throughout the twelve months. As one inch on the barometer corresponds to nearly 1000 feet of altitude, we can see the great error that may be involved when only a few readings are employed in determining the elevation. Altitudes determined from readings taken several times a day for every day in the year for a number of years will certainly not be materially affected by gradient errors.

There are several different coefficients that are used in the barometric formulæ; for example, we have the coefficients, 60,096, 60,384, 60,158.6, and several others differing only a little from these. The formulæ in which these coefficients are used are practically the same. The first value above, Biot and Arago's, and the second, Regnault's, were determined by experiments in the laboratory. The third, Ramond's, was determined by making the results of the formulæ agree with those determined by trigonometrical leveling. In determining this constant Ramond used the results of only eight observa-

tions. This could not, except under the most favorable circumstances, give an accurate result, for reasons already stated.

In order to find which of the various constants comes the nearest filling the requirements, I have taken Laplace's formulæ

$$Z = log\,\frac{h}{H} \times 60158.6\,ft.\,(1+\frac{t+t'-64}{900})\,(1+0.0026\,cos\,2\,L)\left(\frac{1+Z+52252}{20886860}+\frac{h}{10443430}\right)$$

and substituted for the unknown quantities values as given in the U. S. Signal Service Report for 1885, then solved the equation for the constant represented by the term 60158.6 above. I have selected the stations that have been occupied for several years, and only such as have had their altitudes determined by spirit leveling. The stations have also been selected so as to involve all parts of the U. S., and thus meet all the variations due to position. In all, twenty pairs have been used. The stations and the results are given in the following table:

STATIONS.	DIFF. OF ELEV.	COEFFICIENT.
Leavenworth, Kans.—Dodge City, Kans.	1675	59670
Portland, Oregon—Columbus, Ohio.	738	63342
Cleveland, Ohio—Baltimore, Md.	645	57461
Palestine, Tex.—Galveston, Tex.	493	62169
Augusta, Ga.—Atlanta, Ga.	946	59358
San Francisco, Cal.—Cape Mendocino, Cal.	577	59809
Pike's Peak, Col.—Denver, Col.	8840	60865
Rochester, N. Y.—Sandy Hook, N. Y.	598	57982
Mt. Washington, N. H.—Oswego, N. Y.	514	59396
Buffalo, N. Y.—Boston, Mass.	6279	63026
Washington City—Pittsburg, Pa.	668	62215
Rochester, N. Y.—New Haven, Conn.	660	57352
Albany, N. Y.—Buffalo, N. Y.	607	56997
Charlotte, N. C.—Baltimore, Md.	764	61418
Sandy Hook, N. J.—Erie, Pa	653	58648
Charlotte, N. C.—Charleston, S. C.	750	61083
Philadelphia, Pa.—Pittsburg, Pa.	649	58446
Block Island, R. I.—Mt. Washington, N. H.	6252	59575
Knoxville, Tenn.—Memphis, Tenn.	660	62245
Atlanta, Ga.—Charleston, S. C.	1077	61965
Mean		60156

Each coefficient is determined from the mean of all the observations for the year 1885. In computing these coefficients, three were found which diverged widely from the mean, and are not given in the table; they are 73,275, 54,273 and 51,103. These were rejected on the assumption that there is some error in the elevations, or some mistake in the readings, as given in the report. The mean for the twenty stations in the list is 60,156, a result which agrees very nearly with that deduced by Ramond. Thus it would seem that Ramond's coefficient is very nearly correct, although probably a matter of pure accident.

In examining the coefficents in the table, it is surprising to find

so great a variation, since each is the result of about 1095 pairs of observations. For an elevation of one foot a change of 1000 in the coefficient will cause a change in the elevation of .0168 feet. The effect on the computed elevation of a change in the barometric coefficient is nearly proportional to the difference of elevation; hence, for an elevation of 1000 feet, a change of 1000 in the coefficient will make a difference of 16.8 feet in the elevation. The greatest variation from the mean coefficient is 3186 which is equivalent to a change of .053 feet per foot of difference of elevation; or, if the mean coefficient, as determined above, is used, the probable error due to uncertainty in the coefficient will be .022 feet per foot of difference of elevation.

* * * * * * * * * * * *

The above investigation, therefore, shows that the result is liable to a considerable error owing to the variation of the coefficient, and also that for places within the U. S., the constant 60156 will give the best results. This coefficient agrees practically with that used in Guyot's tables (see Smithsonian Miscellaneous Collection, volume 1), and hence those tables are better than those based on any other constant.

SUGAR IN MORTAR.†

I. O. BAKER, Professor of Civil Engineering.

Although saccharine matter has been employed as an ingredient of mortar in India from time immemorial and reference has been made to it by standard authorities, its effect is not generally known, and has attracted considerable attention in England and America during the past year.

Sugar unites with lime, and forms sucrate of lime, a solid which possesses considerable strength, dissolves freely in water, and is acted upon by carbonic acid. All hydraulic cements contain at least 50 per cent. of lime compounds. Hence, if a saccharine substance be added to mortar, the sugar will unite with the lime and form sucrate of lime; the effect of this compound may be an advantage or a disadvantage depending upon attendant conditions. For example, if the mortar is composed of common lime and sand, the sucrate of lime, being stronger than the carbonate, will add to the strength of the mortar; the lime will unite more rapidly with the sugar than with the carbonic acid of the air, and hence the addition of the sugar will also cause the mortar to set more quickly. In India, the practice is to add 1 lb. of the coarsest sugar (or its equivalent in syrup) to each gallon of water with which the mortar is mixed; "this amount of sugar adds one-half to the breaking strength of the mortar and doubles its cohesive strength." It is better to dissolve the sugar in the water instead of mixing it dry with the lime, since some limes in slacking "burn" the sugar, thereby destroying its strengthening effect and also blackening the mortar.

If the mortar is composed of cement and sand, the addition of sugar may increase or decrease the ultimate strength of the mortar depending upon the amount of sugar present and also upon the relative ultimate strength of the compounds formed. The addition of sugar to cement mortar will also accelerate or retard the setting of the cement, depending (1) upon the relative indurating activity of the sucrate and the silicate, and (2) upon the amount of water used, for while the cement is hydraulic the sucrate is non-hydraulic, and hence the former will set in the presence of water but the second will not. Either of these facts accounts for the opposite results ob-

† At the last moment the publication committee was disappointed in not receiving, owing to unavoidable circumstances, three papers which had been promised; one of the articles was to have given an account of some experiments on the effect of sugar in mortar, and, therefore, the committee decided to publish the following extract from Professor Baker's blue-prints on Masonry Construction, although it was neither written for the Club nor read at a meeting of it.

tained from different experiments,* and also for the fact that some experimenters conclude that sugar is of no advantage with the best qualities of Portland cement.†

Sucrate of lime is soluble in water, and hence in time it will be washed out by the rain; therefore, the addition of a saccharine substance to mortar is most beneficial in a dry climate, as India, for example. If lime mortar is used in the interior of thick walls, the addition of a saccharine substance would be beneficial, since mortar thus placed would never‡ become fully saturated with carbonic acid. The compounds of lime with sugar are attacked by the carbonic acid of the air, and hence the strengthening effect of the sugar is not permanent where the mortar is exposed to the weather.

It is highly probable that essentially the effects obtained by mixing sugar with mortar can also be obtained by the use of gum arabic, dextrine, glucose, or starch. The use of such materials in mortar involves some interesting questions; a study of this subject by an engineer-chemist might lead to valuable results.

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* Compare the results in *Engineering News*, vol. 17, p. 6, with those in *Mechanics*, vol. 9, pp. 315-7 (a paper read at the Washington meeting of the American Society of Mechanical Engineers).

† *Engineering News*, vol. 16, p. 333.

‡ Lime mortars have been taken from the walls of ancient buildings which were only 50 to 80 per cent. saturated with carbonic acid after an exposure of nearly 2,000 years.

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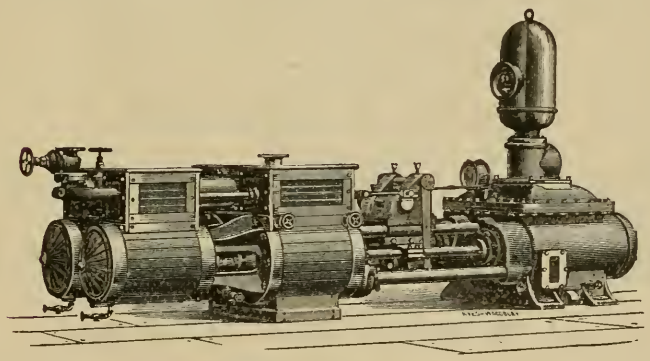

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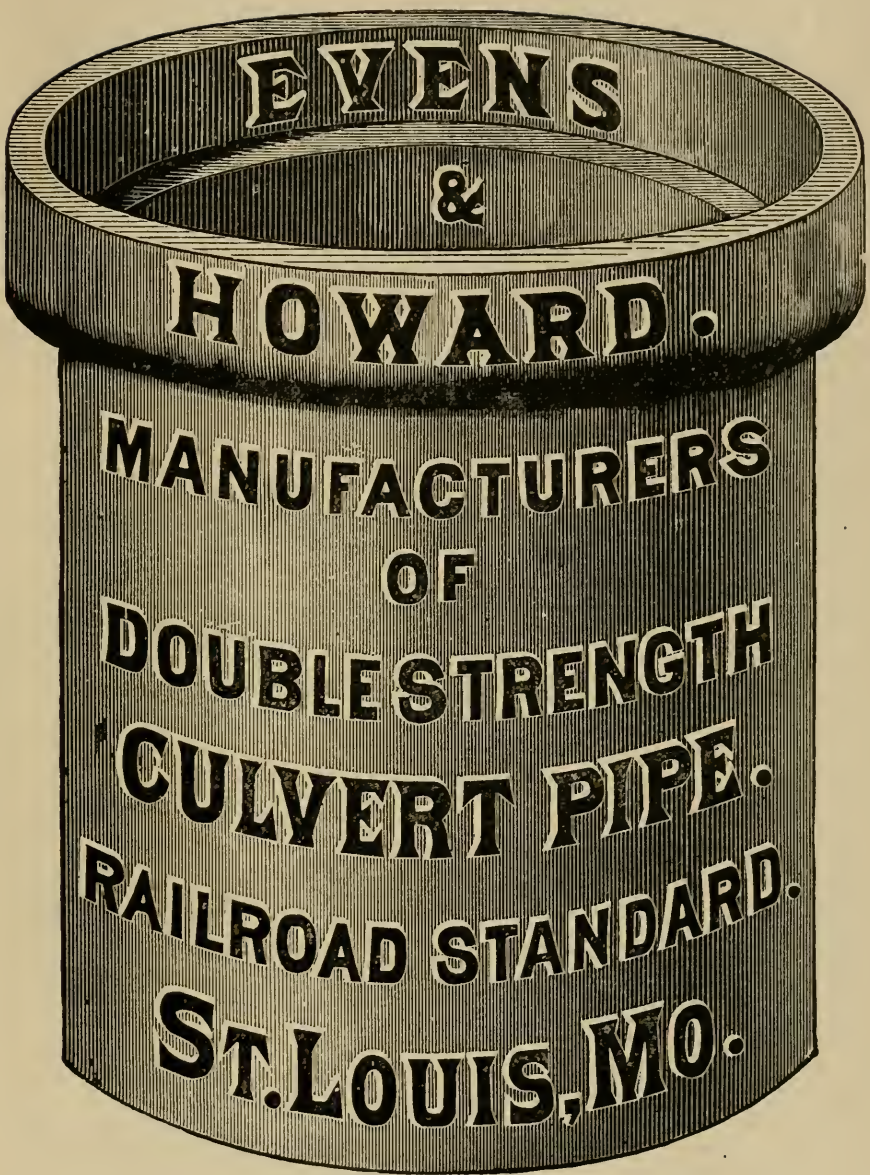

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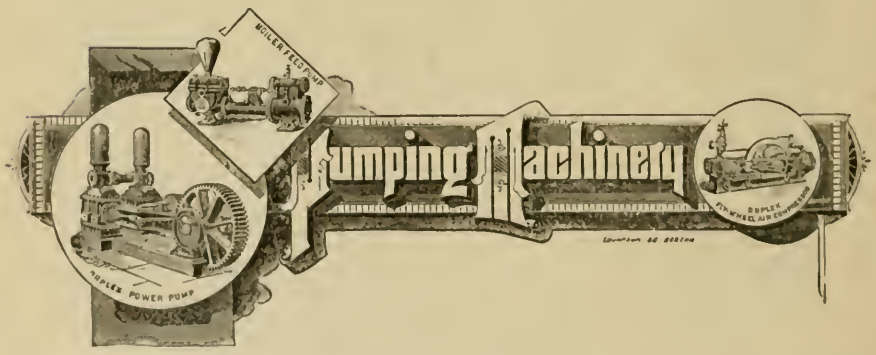

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